1. INTRODUCTION

1.1 Purpose

This report, *FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications* has been prepared by the SAC Joint Venture, under contract to the Federal Emergency Management Agency, to indicate those standards of workmanship for structural steel fabrication and erection deemed necessary to achieve reliably the design performance objectives contained in the set of companion publications prepared under this same contract:

- *FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings,* which provides recommended criteria, supplemental to *FEMA-302, 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures,* for the design and construction of steel moment-frame buildings and provides alternative performance-based design criteria;
- *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings,* which provides recommended methods to evaluate the probable performance of existing steel moment-frame buildings in future earthquakes and to retrofit these buildings for improved performance; and
- *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded, Steel Moment-Frame Buildings*, which provides recommendations for performing postearthquake inspections to detect damage in steel moment-frame buildings following an earthquake, evaluating the damaged buildings to determine their safety in the postearthquake environment, and repairing damaged buildings.

The recommended design criteria contained in these three companion reports are based on the material and workmanship standards contained in this document, which also includes discussion of the basis for the quality control and quality assurance criteria contained in the recommended specifications.

This document has been prepared in two parts.

Part I, Recommended Specifications, provides recommended supplemental requirements, for typical project specifications for structural steel fabrication and erection, which should be included in specifications for the construction of steel moment frames designed for seismic applications. These recommendations have not been subjected to a formal consensus adoption process, nor was formal review or approval obtained from technical committees of the Structural Engineers Association of California (SEAOC). However, these recommendations have received extensive review by practicing engineers, researchers, fabricators and erectors, and the standards of construction indicated by these recommendations were presumed to exist during the development of the design criteria contained in *FEMA-350*, *FEMA-351*, and *FEMA-352*. It is anticipated that these recommendations will be submitted for consideration by the applicable standards committees of the American Institute of Steel Construction (AISC), American Society for Testing and Materials (ASTM), American Society for Nondestructive Testing (ASNT), and American Welding Society (AWS). These organizations are expected to subject these recommendations to a consensus review process, the result of which may be the modification of the industry standard specifications to incorporate these recommendations, perhaps in modified or abbreviated form. In the interim, it is recommended that the applicable portions of these Recommended Specifications be included in construction documents where the design is based on *FEMA-350*, *FEMA-351*, or *FEMA-352*.

Part II, Recommended Quality Assurance Guidelines, has been prepared to provide design professionals, building officials, and contractors with recommended procedures for performing quality control and quality assurance functions in the construction of steel moment frames designed for seismic applications. These recommendations are non-mandatory but are deemed appropriate to achieving the construction standards presumed in the design criteria presented in *FEMA-350*, *FEMA-351* and *FEMA-352*, and may be used as a resource in developing Quality Assurance Plans as required in *FEMA-302*, *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*.

This document is not intended to serve as a complete model for direct adoption as building code provisions, nor is it intended to supplant existing building code, design specification or material specification provisions, except as specifically noted in the text. Because building codes, design specifications, and material specifications are constantly being revised, users of this document are cautioned to review its provisions to verify their suitability and compliance with current governing standards. This document has been prepared using standards in existence or soon to be adopted at the time of publication.

Some of the material contained in this document replicates provisions of existing codes and standards. This has been done for the following reasons: to provide a complete document using a style typical of building project specifications; to highlight critical portions of existing standards that are commonly missed because the contents are contained within larger codes, specifications and standards, and may be atypical of common requirements for general steel construction; and to establish the basis for the provisions themselves.

Detailed explanations of the basis for these recommended specifications and quality assurance guidelines, hereinafter referred to as *Recommendations*, may be found in a series of State of the Art Reports prepared in parallel with these criteria. These reports include:

- *FEMA-355A – State of the Art Report on Base Metals and Fracture*. This report summarizes current knowledge of the properties of structural steels commonly employed in building construction, and the production and service factors that affect these properties.
- *FEMA-355B – State of the Art Report on Welding and Inspection*. This report summarizes current knowledge of the properties of structural welding commonly employed in building construction, the effect of various welding parameters on these properties, and the effectiveness of various inspection methodologies in characterizing the quality of welded construction.

- *FEMA-355C – State of the Art Report on Systems Performance.* This report summarizes an extensive series of analytical investigations into the demands induced in steel moment-frame buildings designed to various criteria, when subjected to a range of different ground motions. The behavior of frames constructed with fully restrained, partially restrained and fracture-vulnerable connections is explored for a series of ground motions, including motion anticipated at near-fault and soft-soil sites.
- *FEMA-355D – State of the Art Report on Connection Performance.* This report summarizes the current state of knowledge of the performance of different types of moment-resisting connections under large inelastic deformation demands. It includes information on fully restrained, partially restrained, and partial strength connections, both welded and bolted, based on laboratory and analytical investigations.
- *FEMA-355E – State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.* This report summarizes investigations of the performance of steel moment-frame buildings in past earthquakes, including the 1995 Kobe, 1994 Northridge, 1992 Landers, 1992 Big Bear, 1989 Loma Prieta and 1971 San Fernando events.
- *FEMA-355F – State of the Art Report on Performance Prediction and Evaluation*. This report describes the results of investigations into the ability of various analytical techniques, commonly used in design, to predict the performance of steel moment-frame buildings subjected to earthquake ground motion. Also presented is the basis for performance-based evaluation procedures contained in the design criteria and guideline documents, *FEMA-350* to *FEMA-353*.

In addition to the recommended design criteria and the State of the Art Reports, a companion document has been prepared for building owners, local community officials and other non-technical audiences who need to understand this issue. *A Policy Guide to Steel Moment-Frame Construction* (*FEMA-354*), addresses the social, economic, and political issues related to the earthquake performance of steel moment-frame buildings. *FEMA-354* also includes discussion of the relative costs and benefits of implementing the recommended criteria.

1.2 Intent

The recommended specifications contained in Part I are primarily intended as a resource document for organizations engaged in the development of industry standard codes and consensus standards for the construction of buildings intended to resist the effects of earthquake ground shaking. These specifications have been developed by professional engineers and researchers, based on the findings of a large multi-year program of investigation and research into the performance of steel moment-frame structures. This process included broad external review by practicing engineers, researchers, fabricators, and the producers of steel and welding consumables, including two workshops convened to obtain direct comment from these stakeholders on the proposed recommendations. These recommended specifications are intended to provide engineers, specifiers, and those involved in the development of project specifications with guidance in writing provisions appropriate for their particular projects. The provisions of this document have been established to provide a concise, detailed set of requirements for steel moment-frame construction in seismic applications. Currently, there are numerous and widely diverse requirements being specified by practicing engineers and building authorities for steel moment-frame construction in seismic applications. These *Recommendations* provide a basis for standardization of those requirements within the industry, consistent with design recommendations developed by this project.

The recommended quality assurance guidelines contained in Part II are intended to provide engineers, building officials, inspection agencies and contractors with an understanding of the important features of a quality control and quality assurance program for steel moment-frame construction for seismic applications. It provides information and recommended guidelines relative to the appropriate qualifications and roles of the various parties engaged in the construction of these structures. In addition, it provides a recommended system of classification for welded joints in seismic applications that may be used to select and specify appropriate quality control and quality assurance measures for these joints. This system has been used to develop the recommended quality assurance categories referenced in the design criteria documents produced by this project, including *FEMA-350*, *FEMA-351* and *FEMA-352*. This same classification system is used in Part I of these *Recommendations* to determine the applicable recommended quality control and quality assurance specification provisions.

1.3 Background

For many years, the basic intent of the building code seismic provisions has been to provide buildings with an ability to withstand intense ground shaking without collapse, but potentially with some significant structural damage. In order to accomplish this, one of the basic principles inherent in modern code provisions is to encourage the use of building configurations, structural systems, materials and details that are capable of ductile behavior. A structure is said to behave in a ductile manner if it is capable of withstanding large inelastic deformations without significant degradation in strength, and without the development of instability and collapse. The design forces specified by building codes for particular structural systems are related to the amount of ductility the system is deemed to possess. Generally, structural systems with more ductility are designed for lower forces than less ductile systems, as ductile systems are deemed capable of resisting demands that are significantly greater than their elastic strength limit. Starting in the 1960s, engineers began to regard welded steel moment-frame buildings as being among the most ductile systems contained in the building code. Many engineers believed that steel moment-frame buildings were essentially invulnerable to earthquake-induced structural damage and thought that should such damage occur, it would be limited to ductile yielding of members and connections. Earthquake-induced collapse was not believed possible. Partly as a result of this belief, many large industrial, commercial and institutional structures employing steel moment-frame systems were constructed, particularly in the western United States.

The Northridge earthquake of January 17, 1994 challenged this paradigm. Following that earthquake, a number of steel moment-frame buildings were found to have experienced brittle fractures of beam-to-column connections. The damaged buildings had heights ranging from one story to 26 stories, and a range of ages spanning from buildings as old as 30 years to structures just being erected at the time of the earthquake. The damaged buildings were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few buildings were located on sites that experienced the strongest ground shaking, damage to buildings on these sites was extensive. Discovery of these unanticipated brittle fractures of framing connections, often with little associated architectural damage to the buildings, was alarming to engineers and the building industry. The discovery also caused some concern that similar, but undiscovered, damage may have occurred in other buildings affected by past earthquakes. Later investigations confirmed such damage in a limited number of buildings affected by the 1992 Landers, 1992 Big Bear and 1989 Loma Prieta earthquakes.

In general, steel moment-frame buildings damaged by the 1994 Northridge earthquake met the basic intent of the building codes. That is, they experienced limited structural damage, but did not collapse. However, the structures did not behave as anticipated and significant economic losses occurred as a result of the connection damage, in some cases, in buildings that had experienced ground shaking less severe than the design level. These losses included direct costs associated with the investigation and repair of this damage as well as indirect losses relating to the temporary, and in a few cases, long-term, loss of use of space within damaged buildings.

Steel moment-frame buildings are anticipated to develop their ductility through the development of yielding in beam-column assemblies at the beam-column connections. This yielding may take the form of plastic hinging in the beams (or, less desirably, in the columns), plastic shear deformation in the column panel zones, or through a combination of these mechanisms. It was believed that the typical connection employed in steel moment-frame construction, shown in Figure 1-1, was capable of developing large plastic rotations, on the order of 0.02 radians or larger, without significant strength degradation.

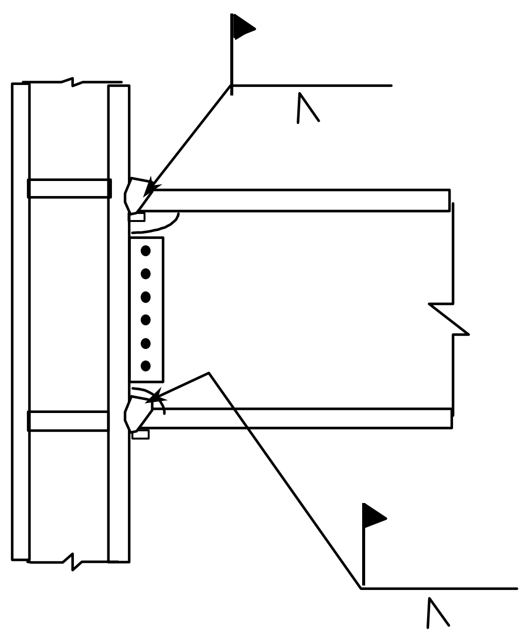


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

Observation of damage sustained by buildings in the 1994 Northridge earthquake indicated that, contrary to the intended behavior, in many cases, brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained essentially elastic. Typically, but not always, fractures initiated at, or near, the complete joint penetration groove weld between the beam bottom flange and column flange (Figure 1-2). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

In some cases, the fractures progressed completely through the thickness of the weld, and when fire protective finishes were removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-3a). Other fracture patterns also developed. In some cases, the fracture developed into a crack of the column flange material behind the complete joint penetration groove weld (Figure 1-3b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the

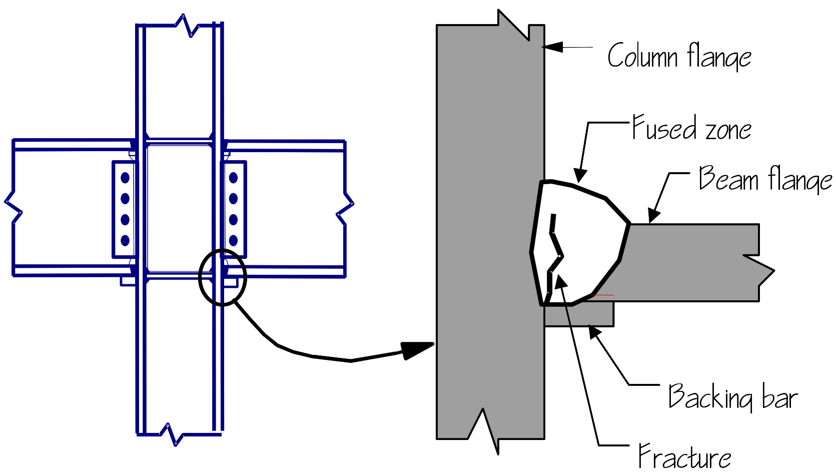
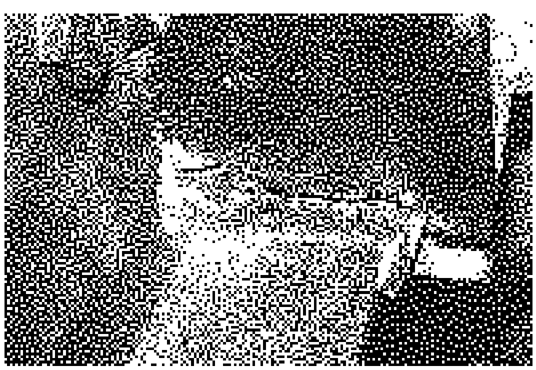
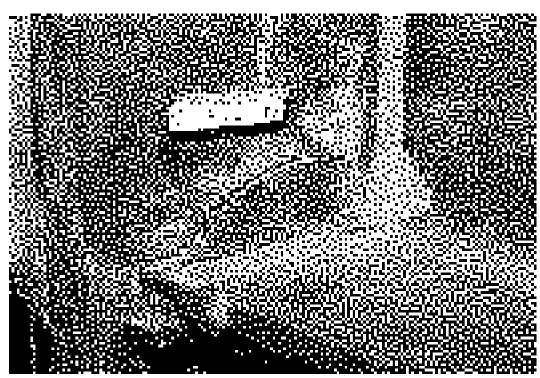


Figure 1-2 Common Zone of Fracture Initiation in Beam-Column Connection



a. Fracture at Fused Zone

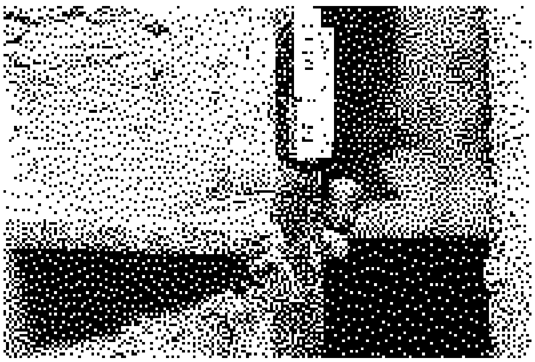


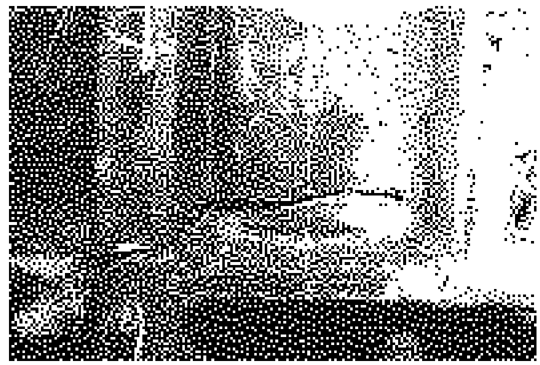
b. Column Flange "Divot" Fracture

Figure 1-3 Fractures of Beam to Column Joints

remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

A number of fractures progressed completely through the column flange, along a near-horizontal plane that aligns approximately with the beam lower flange (Figure 1-4a). In some cases, these fractures extended into the column web and progressed across the panel zone (Figure 1-4b). Investigators have reported some instances where columns fractured entirely across the section.





a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-4 Column Fractures

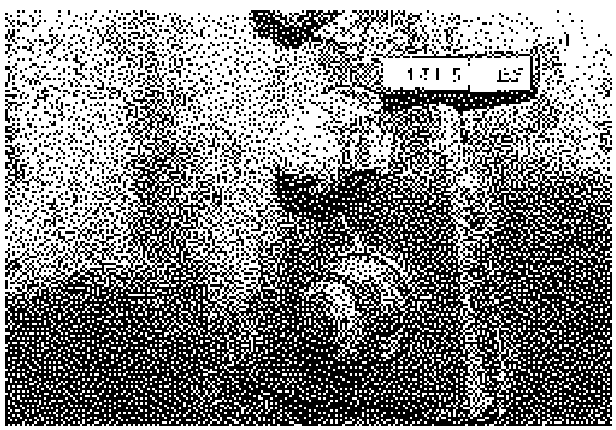


Figure 1-5 Vertical Fracture through Beam Shear Plate Connection

Once such fractures have occurred, the beam-column connection has experienced a significant loss of flexural rigidity and strength to resist those loads that tend to open the crack. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. However, in providing this residual strength and stiffness, the bolted web connections can themselves be subject to failures. These include fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-5).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or damage to architectural elements, making reliable postearthquake damage evaluations difficult. In order to determine reliably if a building has sustained connection damage it is necessary to remove architectural finishes and fireproofing, and perform detailed inspections of the connections. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. At least one steel moment-frame building sustained so much damage that it was deemed more practical to demolish the building than to repair it.

Initially, the steel construction industry took the lead in investigating the causes of this unanticipated damage and in developing design recommendations. The American Institute of Steel Construction (AISC) convened a special task committee in March, 1994 to collect and disseminate available information on the extent of the problem (AISC, 1994a). In addition, together with a private party engaged in the construction of a major steel building at the time of the earthquake, the American Institute of Steel Construction (AISC) participated in sponsoring a limited series of tests of alternative connection details at the University of Texas at Austin (AISC, 1994b). The American Welding Society (AWS) also convened a special task group to investigate the extent that the damage related to welding practice and to determine if changes to the welding code were appropriate (AWS, 1995).

In September, 1994, the SAC Joint Venture, AISC, the American Iron and Steel Institute and National Institute of Standards and Technology jointly convened an international workshop (SAC, 1994) in Los Angeles to coordinate the efforts of the various participants and to lay the foundation for systematic investigation and resolution of the problem. Following this workshop, FEMA entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused studies of the seismic performance of steel moment-frame buildings and to develop recommendations for professional practice (Phase I of SAC Steel Project). Specifically, these recommendations were intended to address the following: the inspection of earthquake-affected buildings to determine if they had sustained significant damage; the repair of damaged buildings; the upgrade of existing buildings to improve their probable future performance; and the design of new structures to provide reliable seismic performance.

During the first half of 1995, an intensive program of research was conducted to explore more definitively the pertinent issues. This research included literature surveys, data collection on affected structures, statistical evaluation of the collected data, analytical studies of damaged and undamaged buildings, and laboratory testing of a series of full-scale beam-column assemblies representing typical pre-Northridge design and construction practice as well as various repair, upgrade and alternative design details. The findings of these tasks formed the basis for the development of *FEMA-267 – Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures*, which was published in August, 1995. *FEMA-267* provided the first definitive, albeit interim, recommendations for practice, following the discovery of connection damage in the 1994 Northridge earthquake.

In September 1995 the SAC Joint Venture entered into a contractual agreement with FEMA to conduct Phase II of the SAC Steel Project. Under Phase II, SAC continued its extensive problem-focused study of the performance of moment resisting steel frames and connections of various configurations, with the ultimate goal of develop seismic design criteria for steel construction. This work has included: extensive analyses of buildings; detailed finite element and fracture mechanics investigations of various connections to identify the effects of connection configuration, material strength, and toughness and weld joint quality on connection behavior; as well as more than 120 full-scale tests of connection assemblies. As a result of these studies, and independent research conducted by others, it is now known that the typical moment-resisting connection detail employed in steel moment-frame construction prior to the 1994 Northridge earthquake, and depicted in Figure 1-1, had a number of features that rendered it inherently susceptible to brittle fracture. These included the following:

- The most severe stresses in the connection assembly occur where the beam joins to the column. Unfortunately, this is also the weakest location in the assembly. At this location, bending moments and shear forces in the beam must be transferred to the column through the combined action of the welded joints between the beam flanges and column flanges and the shear tab. The combined section properties of these elements, for example the cross sectional area and section modulus, are typically less than those of the connected beam. As a result, stresses are locally intensified at this location.

- The joint between the bottom beam flange and the column flange is typically made as a downhand field weld, often by a welder sitting on top of the beam top flange, in a so-called "wildcat" position. To make the weld from this position each pass must be interrupted at the beam web, with either a start or stop of the weld at this location. This welding technique often results in poor quality welding at this critical location, with slag inclusions, lack of fusion and other defects. These defects can serve as crack initiators, when the connection is subjected to severe stress and strain demands.

- The basic configuration of the connection makes it difficult to detect hidden defects at the root of the welded beam-flange-to-column-flange joints. The backing bar, which was typically left in place following weld completion, restricts visual observation of the weld root. Therefore, the primary method of detecting defects in these joints is through the use of ultrasonic testing. However, the geometry of the connection also makes it very difficult for ultrasonic testing to detect flaws reliably at the bottom beam flange weld root, particularly at the center of the joint, at the beam web. As a result, many of these welded joints have undetected significant defects that can serve as crack initiators.

- Although typical design models for this connection assume that nearly all beam flexural stresses are transmitted by the flanges and all beam shear forces by the web, in reality, due to boundary conditions imposed by column deformations, the beam flanges at the connection carry a significant amount of the beam shear. This results in significant flexural stresses on the beam flange at the face of the column, and also induces large secondary stresses in the welded joint. Some of the earliest investigations of these stress concentration effects in the welded joint were conducted by Richard, et al. (1995). The stress concentrations resulting from this effect resulted in severe strength demands at the root of the complete joint penetration welds between the beam flanges and column flanges, a region that often includes significant discontinuities and slag inclusions, which are ready crack initiators.

- In order that the welding of the beam flanges to the column flanges be continuous across the thickness of the beam web, this detail incorporates a series of weld access holes in the beam web, at the beam flanges. Depending on their geometry, severe strain concentrations can occur in the beam flange at the toe of these weld access holes. These strain concentrations can result in low-cycle fatigue and the initiation of ductile tearing of the beam flanges after only a few cycles of moderate plastic deformation. Under large plastic flexural demands, these ductile tears can quickly become unstable and propagate across the beam flange.

- Steel material at the center of the beam-flange-to-column-flange joint is restrained from movement, particularly in connections of heavy sections with thick column flanges. This condition of restraint inhibits the development of yielding at this location, resulting in locally high stresses on the welded joint, which exacerbates the tendency to initiate fractures at defects in the welded joints.

- Design practice in the period 1985-1994 encouraged design of these connections with relatively weak panel zones. In connections with excessively weak panel zones, inelastic behavior of the assembly is dominated by shear deformation of the panel zone. This panel zone shear deformation results in a local kinking of the column flanges adjacent to the beam-flange-to-column-flange joint, and further increases the stress and strain demands in this sensitive region.

In addition to the above, additional conditions contributed significantly to the vulnerability of connections constructed prior to 1994.

- In the mid-1960s, the construction industry moved to the use of the semi-automatic, self-shielded, flux-cored arc welding process (FCAW-S) for making the joints of these connections. The welding consumables that building erectors most commonly used inherently produced welds with very low toughness. The toughness of this material could be further compromised by excessive deposition rates, which unfortunately were commonly employed by welders. As a result, brittle fractures could initiate in welds with large defects, at stresses approximating the yield strength of the beam steel, precluding the development of ductile behavior.

- Early steel moment frames tended to be highly redundant and nearly every beam-column joint was constructed to behave as part of the lateral-force-resisting system. As a result, member sizes in these early frames were small and much of the early acceptance testing of this typical detail was conducted with specimens constructed of small framing members. As the cost of construction labor increased, the industry found that it was more economical to construct steel moment-frame buildings by moment-connecting a relatively small percentage of the beams and columns and by using larger members for these few moment-connected elements. The amount of strain demand placed on the connection elements of a steel moment frame is related to the span-to-depth ratio of the member. Therefore, as member sizes increased, strain demands on the welded connections also increased, making the connections more susceptible to brittle behavior.

- In the 1960s and 1970s, when much of the initial research on steel moment-frame construction was performed, beams were commonly fabricated using A36 material. In the

1980s, many steel mills adopted more modern production processes, including the use of scrap-based production. Steels produced by these more modern processes tended to include micro-alloying elements that increased the strength of the materials so that despite the common specification of A36 material for beams, many beams actually had yield strengths that approximated or exceeded that required for grade 50 material. As a result of this increase in base metal yield strength, the weld metal in the beam-flange-to-column-flange joints became under-matched, potentially contributing to its vulnerability.

At this time, it is clear that in order to obtain reliable ductile behavior of steel moment-frame construction a number of changes to past practices in design, materials, fabrication, erection and quality assurance are necessary. The recommended criteria contained in this document, and the companion publications, are based on an extensive program of research into materials, welding technology, inspection methods, frame system behavior, and laboratory and analytical investigations of different connection details. The guidelines presented herein are believed to be capable of addressing the vulnerabilities identified above and providing for frames capable of more reliable performance in response to earthquake ground shaking.

1.4 Application

These *Recommendations* supersede the quality assurance guidelines contained in *FEMA-267*, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, and the *Interim Guidelines Advisories*, *FEMA-267A* and *FEMA-267B*. It is intended to be used in coordination with and in supplement to the locally applicable building code and those national standards referenced by the building code. Building codes are living documents and are revised on a periodic basis. These *Recommendations* have been prepared based on the provisions contained in the 1997 *FEMA-302 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, the 1997 *AISC Seismic Specification* (AISC, 1997) and the 1998 *AWS D1.1 Structural Welding Code - Steel*, as these documents form the basis for the 2000 edition of the *International Building Code*. Some users may wish to apply the recommendations contained herein to specific engineering projects, prior to the adoption of these recommendations by future codes and standards. Such users are cautioned to consider carefully any differences between the aforementioned documents and those actually enforced by the building department having jurisdiction for a specific project and to adjust these recommendations accordingly. These users are also warned that these recommendations have not undergone a consensus adoption process. Users should thoroughly acquaint themselves with the technical data upon which these recommendations are based and exercise their own independent engineering judgment prior to implementing these recommendations in practice.

1.5 Overview

The following is an overview of the general contents of chapters contained in these *Recommendations*, and their intended use. Many of the specification provisions contained in Part I are already included in industry standard specification and building code requirements.

When this document is used as a basis for the development of project specifications, users should include those paragraphs specifically applicable to the individual project. This includes provisions of a general nature that may be necessary for the overall project, as well as specific provisions applicable to the connection systems and details being utilized. The recommended specifications contained in Part I are not intended as a stand-alone specification. They are intended to be used as a supplement to, and in coordination with, a complete specification for structural steel construction.

The user should also review the incorporated provisions for compliance with local building code requirements, particularly in those jurisdictions that may not have adopted the latest model specifications, and those with specific provisions not found in model specifications. In some cases, a variance may be needed to utilize these provisions where revisions to the existing codes and standards are recommended.

Recommendations for new requirements applicable to the construction of seismic-force-resisting systems are indicated in these *Recommendations* by underlined text.

Part I – Recommended Specifications

- **Chapter 1: General**. This chapter provides definitions for use throughout Part I. It also contains provisions pertaining to submittal of information regarding material, material certifications, procedures, personnel records, quality control plan, and samples. The use of a pre-fabrication, pre-erection conference is also established.
- **Chapter 2: Products**. This chapter lists the applicable material specifications, both generic and specific, for structural steel, welding material, bolting material, and shear connectors.
- **Chapter 3: Execution**. This chapter provides generic and specific recommendations for the fabrication and erection of the structural steel frames, bolting, and welding. Included are several specific recommendations regarding welding operations that are not currently in the welding codes.
- **Chapter 4: Welded Joint Details**. This chapter provides detailing and welding recommendations for specific types of welded joints, including backing bars, weld tabs, reinforcing fillet welds, weld access holes, web connections, doubler plates, continuity plates, cover-plated connections, welded overlay connections, and haunched connections.
- **Chapter 5: Fabrication Details**. This chapter provides detailing and quality recommendations for steel fabrication, welding, cutting, bolting, and repairs. Quality recommendations for beam-flange-to-column-flange moment-connection welds are provided in this chapter.
- **Chapter 6: Quality Control and Quality Assurance**. This chapter lists governing specifications and practices, the requirements and recommendations for a Written Practice for quality assurance and nondestructive testing, inspector qualifications, nondestructive testing technician qualifications, and a detailed list of quality control and quality assurance tasks for welding, bolting, and shear connector inspection. Specific recommendations for nondestructive testing for various joints are included.

- **Appendices.** Several appendices are provided with details for various recommended test procedures for welding material, welding personnel qualification, and nondestructive testing technician qualification. Recommended provisions for magnetic particle testing procedures are also provided.

Part II – Quality Assurance Guidelines

- **Chapter 1: General.** This chapter provides discussion of various terms relating to the process of ensuring that workmanship and materials conform to the applicable standards. In addition it describes the various phases of the quality process and the recommended role of various participants in these phases.
- **Chapter 2: Contractor Qualifications and Quality Tasks.** This chapter describes methods for determining whether contractors have adequate qualifications to perform the work, and also provides recommended contractor responsibilities in the quality process.
- **Chapter 3: Quality Assurance Agency Qualifications and Quality Assurance Tasks.** This chapter describes methods for determining whether inspection agencies and testing laboratories that perform quality control and assurance tasks, and their personnel engaged in these tasks, have adequate qualifications for this work. It also includes recommendations for inspection agency work scope as part of the quality process.
- **Chapter 4: Structural Steel.** This chapter provides recommendations for procedures to ensure that structural steel materials meet the applicable standards. Checklists for assisting in the verification of steel quality are included.
- **Chapter 5: Welding.** This chapter presents recommended methods for ensuring that welded joints meet the applicable materials and workmanship standards. Included in this chapter is a description of the basis for determining the quality assurance category for a welded joint, used in Part I of these *Recommendations* as an index to recommended quality assurance measures.
- **Chapter 6: Bolting.** This chapter presents recommended procedures for determining whether bolted joints meet the applicable materials and workmanship requirements.
- **References, Bibliography, and Acronyms.**

> *Commentary: The Recommended Specifications and Quality Assurance Guidelines contained in these Recommendations are applicable to the construction of moment-resisting frames used as the seismic-force-resisting elements of buildings and other structures, designed in accordance with the recommendations of FEMA-350, FEMA-351 and FEMA-352. As used in these Recommendations, the terms "steel structure" or "steel framing" apply only to the structural steel elements of the seismic-force-resisting system as defined in FEMA-302. The quality assurance guidelines and quality control guidelines contained in these Recommendations could be extended to the construction of other types of seismic-force-resisting systems and even to elements of structures that are not intended to be part of the seismic-force-resisting system. However,*

extension of these recommendations to these other construction types and elements could result in significant construction cost premiums. These Recommendations are deemed appropriate to application to the construction, upgrade and repair of Special Moment Frames and Ordinary Moment Frames used in seismic-force-resisting systems due to the severe service conditions anticipated for these frames and the proven sensitivity of their performance to construction quality. Such determination has not been made for other elements or systems. Further discussion may be found in FEMA-354, A Policy Guide to Steel Moment-Frame Construction.